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16. Abstract

Highway safety is an ongoing concern to the Texas Department of Transportation (TxDOT). As part of its proactive commitment to improving highway safety, TxDOT is moving toward including quantitative safety analyses earlier in the project development process. The objectives of this research project are: (1) the development of safety design guidelines and evaluation tools to be used by TxDOT designers, and (2) the production of a plan for the incorporation of these guidelines and tools in the planning and design stages of the project development process.

This document describes a procedure for using accident modification factors in the highway design process. Application of the procedure entails the use of several factors, where each factor addresses one specific design element (such as lane width, shoulder width, curve radius, etc.). Collectively, the factors can be used to estimate the effect of a change in one or more design elements. The procedure can be used to evaluate the safety benefits associated with alternative geometric designs.

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PROCEDURE FOR USING ACCIDENT MODIFICATION FACTORS IN THE HIGHWAY DESIGN PROCESS

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data published herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration (FHWA) and/or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation. It is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was James Bonneson, P.E. #67178.

NOTICE

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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- Mr. Richard Harper (TxDOT);
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CHAPTER 1. INTRODUCTION

OVERVIEW

This report describes a procedure for estimating the safety associated with a specified roadway segment or intersection, as may be influenced by their design elements. In this application, "safety" is defined to be the expected injury (plus fatal) crash frequency for the segment or intersection. The procedure combines the best elements of regression models with before-after study results and historical crash data to yield a "best estimate" of safety. It is based on, and closely follows, the procedure developed by Harwood et al. (*1*). It is envisioned that the procedure will be used with the tools provided in the *Interim Roadway Safety Design Workbook* (*Workbook*) (*2*) to evaluate alternative design configurations.

Conditions Where the Procedure May Be Helpful

The safety evaluation procedure described in this report can be used throughout the geometric design process. However, the insights provided through its use will be most helpful in situations where the choice among design elements is not obvious or the trade-offs are not readily apparent (e.g., where atypical conditions exist, the design is complex, or construction costs are high). In this manner, the procedure will facilitate the thoughtful and balanced consideration of both safety and operational benefits as well as the costs associated with construction, maintenance, and environmental impacts.

Experience indicates that several "key" design elements oftentimes have a relatively important relationship with safety. These design elements can be characterized as being: (1) associated with the "controlling criteria" that dictate the need for a design exception or have a known effect on safety, and (2) frequently used in situations where atypical conditions exist, the design is complex, or construction costs are high. The controlling criteria for new location and reconstruction projects are identified in the *Roadway Design Manual* (*3*) and include:

- design speed,
- lane width,
- shoulder width,
- bridge width,
- structural capacity,
- horizontal alignment,
- vertical alignment,
- grade,
- stopping sight distance,
- cross slope,
- superelevation, and
- vertical clearance.

Additional design elements that may also be considered as "key" because of their known effect on safety include: a turn bay at an intersection, a median treatment, and the clear zone (i.e., horizontal clearance). For non-key design elements, the traditional design process (i.e., compliance with design criteria and warrants) will likely provide an acceptable level of safety.

The implementation of this procedure will add time to the design process. However, by limiting the evaluation of safety to primarily "key" design elements and complex conditions, it is hoped that the additional time required will be kept to a minimum and incurred only where it is likely to provide some return in terms of improved safety, lower construction cost, or both. This added time represents an immediate and direct cost to the design process. However, it also represents a more cost-effective approach to design because additional benefit will be derived through fewer crashes and lower construction costs (by not over-designing some design elements).

Emphasis on Injury Crash Frequency

Differences in crash reporting threshold among agencies can introduce uncertainty in crash data analysis and regional comparison of crash trends. A majority of the crashes that often go unreported (or, if reported, not filed by the agency) are those identified as "property-damage-only." In contrast, crashes with an injury or fatality tend to be more consistently reported across jurisdictions. Thus, safety relationships tend to be more transferrable among jurisdictions when they are developed using only injury (plus fatal) crash data. In recognition of this benefit, this document is focused on models applicable to injury (plus fatal) crashes. Unless explicitly stated otherwise, all references to "crash frequency" refer to *injury (plus fatal)* crash frequency.

An injury crash is a crash wherein one or more of the persons involved is injured. The injury severity is reported as "possible," "non-incapacitating," or "incapacitating."

ORGANIZATION

This report consists of three main parts. Each of these parts is presented as a separate chapter. The first part, presented in Chapter 2, describes the methodology that underlies the safety prediction procedure. It presents the basic concepts, the rationale, and the models that are used in the procedure. The second part, presented in Chapter 3, describes the procedure for estimating the safety of a specific roadway segment, or an entire roadway section. The last part, presented in Chapter 4, describes several models that have been developed since the publication of the *Workbook*. They are intended to replace their counterparts published in the *Workbook*.

CHAPTER 2. SAFETY PREDICTION METHODOLOGY

OVERVIEW

This chapter describes a methodology for estimating the safety of a roadway facility component (i.e., roadway segment, intersection, or interchange ramp). In many cases, the safety of an individual segment or intersection is of interest, and it is the sole subject of evaluation. In other cases, the safety of an entire roadway "section" is of interest, where a section typically consists of one or more segments and intersections. In this case, the procedure is separately applied to each segment and intersection that comprises the section and the estimated safety effects summed to yield an estimate of roadway section safety.

The chapter consists of four main parts. The first part describes the safety prediction model as well as the role of the base model and various accident modification factors (AMFs) in it. The second part describes how a base model can be used to predict the expected annual crash frequency for a typical roadway segment, intersection, or interchange ramp. The third part describes how AMFs for various design-related elements can be used to adjust the estimate obtained from the base model to yield an expected crash frequency consistent with the geometric and traffic control characteristics of a given segment, intersection, or ramp. The last part describes how the safety prediction model can be modified to include information about the crash history for a project location. Inclusion of this information increases the accuracy of the estimated expected crash frequency.

SAFETY PREDICTION MODEL

The expected crash frequency for a facility component with specified attributes is computed using a safety prediction model. This model represents the combination of a "base" model and one or more AMFs. The base model is used to estimate the expected crash frequency for a typical segment or intersection. The AMFs are used to adjust the base estimate when the attributes of the specific component are not considered typical. The basic form of the safety prediction model is shown in Equation 1.

$$E[N] = E[N]_b \times AMF_1 \times AMF_2 \ldots \times AMF_n \qquad \textbf{(1)}$$

where,
$E[N]$ = expected crash frequency, crashes/yr;
$E[N]_b$ = expected base crash frequency, crashes/yr; and
AMF_i = accident modification factor for geometry or traffic control variable i (i = 1, 2, ..., n).

Equation 1 yields the expected crash frequency for one facility component. It would be used to compute a similar estimate for each component of interest. If it is used for all m components that comprise a roadway section, then the expected crash frequency for the entire section is computed as:

$$E[N]_s = \sum_{j=1}^{m} E[N]_j \qquad \textbf{(2)}$$

where,

$E[N]_s$ = expected crash frequency for entire roadway section, crashes/yr; and

$E[N]_j$ = expected base crash frequency for segment (or intersection) j, crashes/yr.

To illustrate the use of Equation 2, consider a section of two-lane highway that consists of two intersections and a 0.5 mi segment of highway between the two intersections. Equation 1 indicates that the expected crash frequency for one intersection is 1.5 crashes/yr, and that for the other intersection is 2.2 crashes/yr. Application of Equation 1 to the highway segment indicates that its expected crash frequency is 0.3 crashes per year. The expected crash frequency for the entire highway section is 4.0 crashes/yr (= 1.5 + 2.2 + 0.3).

BASE MODEL

This part of the chapter describes how a base model is used to predict the expected annual crash frequency for a typical roadway segment, intersection, or interchange ramp. A generalized base model for each facility component is summarized in the three subsequent sections. Calibrated versions of these models are provided in the *Workbook* (*2*).

Road Segments

The generalized base model form for roadway segments is:

$$E[N]_b = a \; ADT^{\,b} \; L \; f \tag{3}$$

where,

$E[N]_b$ = expected base crash frequency, crashes/yr;

a, b = calibration coefficients;

ADT = average daily traffic volume, veh/d;

L = highway segment length, mi; and

f = local calibration factor.

The segment base models in the *Workbook* have coefficient b equal to 1.0. When this coefficient is equal to 1.0, then the coefficient a in Equation 3 is effectively equal to the crash rate, with units of "crashes per million vehicle-miles." Chapter 4 describes a replacement base model for rural two-lane highways. This model should be used instead of the one published in the *Workbook*.

The local calibration factor f is included in the base model to allow it to be calibrated to local conditions. Various calibration methods are available; however, the one described by Harwood et al. (*1*) does not require sophisticated statistical techniques and is recommended for practical applications. The models described in Chapter 4 have been calibrated with data from Texas and should not require further calibration.

Intersections

Two variations of Equation 3 are used to estimate the expected crash frequency for intersections, where the AMFs included in it are applicable to intersection design elements. A common form for the intersection base model is:

$$E[N]_b = a\ ADT_{major}^{b_1}\ ADT_{minor}^{b_2}\ f \quad \textbf{(4)}$$

where,
a, b_1, b_2 = calibration coefficients;
ADT_{major} = average daily traffic volume on the major road, veh/d; and
ADT_{minor} = average daily traffic volume on the minor road, veh/d.

This model form predicts 0.0 crashes when either *ADT* variable is equal to 0.0. This boundary condition is illogical because some types of crashes (e.g., rear-end) are still likely to occur when one of the *ADT* variables is 0.0 and the other is nonzero. An alternative form for the intersection base model that does not share this limitation is:

$$E[N]_b = a\ (ADT_{major} + ADT_{minor})^b\ f \quad \textbf{(5)}$$

The intersection base models in the *Workbook* use the form shown in Equation 5 with the coefficient b equal to 1.0. In this situation, the coefficient a is effectively equal to the crash rate, with units of "crashes per million entering vehicles."

The selected intersection base model form is typically used for all intersection configurations (i.e., three-leg, four-leg, etc.) and control types (i.e., signalized, two-way stop control, etc.). However, the model is separately calibrated for each configuration and control type combination. This approach yields unique calibration coefficients for each combination.

Interchange Ramps

The generalized base model form for interchange ramps is:

$$E[N]_b = a\ ADT_r^b\ f \quad \textbf{(6)}$$

where,
$E[N]_b$ = expected base crash frequency, crashes/yr; and
ADT_r = average daily traffic volume on the ramp, veh/d.

The interchange ramp base model in the *Workbook* uses the form shown in Equation 6 with the coefficient b equal to 1.0. In this situation, the coefficient a is effectively equal to the crash rate, with units of "crashes per million vehicles." Conceivably, ramp length could be added to Equation 6; however, research to date has not demonstrated this relationship.

ACCIDENT MODIFICATION FACTOR

AMFs are used to adjust the expected crash frequency estimate obtained from the base model. As suggested by Equation 1, this adjustment is multiplicative. The adjustment is needed when a facility component of interest has one or more characteristics that are atypical. Through this adjustment, the resulting expected crash frequency more accurately reflects the geometric and traffic control characteristics for the given component.

By definition, an AMF represents the relative change that occurs in crash frequency when a particular geometric design component is added or removed, or when a design element is changed in size. An AMF is sometimes calculated as the quotient of the expected crash frequency during the "after" period divided by the expected crash frequency during the "before" period, where the change in design exists only during the after period. AMFs typically range in value from 0.5 to 2.0, with a value of 1.0 representing no effect of the design change. AMFs less than 1.0 indicate that the design change is associated with fewer crashes.

The term AMF is a relatively new term that is closely related to the more familiar crash reduction factor (CRF) used in various hazard elimination programs. Mathematically, the relationship between the AMF and CRF is defined as:

$$AMF = 1 - CRF \tag{7}$$

In spite of their mathematical similarities, the techniques used to quantify the two factors using crash data are quite different. CRFs have historically been developed using "simple" before-after studies that to not control for various sources of bias. In fact, research has shown that the use of the simple before-after study method to develop a CRF often leads to biased values that overstate the true effectiveness of an improvement *(4)*. Recent advances in statistical analysis methods have made it possible to minimize these sources of bias. Researchers using these new methods report their findings using the term "AMF" to avoid any confusion. These new methods were used to develop the AMFs described in this chapter.

Table 1 lists the AMFs provided in the *Workbook*. These AMFs were derived from research findings documented in the literature. They reflect the influence of many, but not all, design elements. The AMFs provided in the *Workbook* were developed to have a value of 1.0 when used to evaluate roadways with typical design and traffic characteristics, as defined by Texas design practice. A table of "base conditions" is provided in each chapter of the *Workbook* to identify these typical characteristics.

Table 1. AMFs Provided in the *Workbook*.

Facility Component	Application	Accident Modification Factor	
Freeway	Geometric design	Grade Outside shoulder width Median width	Lane width Inside shoulder width Shoulder rumble strips
	Roadside design	Utility pole offset	
Rural highway	Geometric design	Horizontal curve radius Grade Outside shoulder width Median width Centerline rumble strip Superelevation	Spiral transition curve Lane width Inside shoulder width Shoulder rumble strips TWLTL median type [1] Passing lane
	Roadside design	Horizontal clearance Utility pole offset	Side slope Bridge width
	Access control	Driveway density	
Urban street	Geometric design	Horizontal curve radius Shoulder width TWLTL median type	Lane width Median width Curb parking
	Roadside design	Utility pole offset	
	Access control	Driveway density	
	Street environment	Truck presence	
Interchange ramp	Geometric design	Only base models for various ramp configurations are available.	
Rural intersections - signalized	Geometric design	Left-turn lane Number of lanes	Right-turn lane Alignment skew angle
	Access control	Driveway frequency	
	Other	Truck presence	
Rural intersections-unsignalized	Geometric design	Left-turn lane Number of lanes Median presence Intersection sight distance	Right-turn lane Shoulder width Alignment skew angle
	Access control	Driveway frequency	
	Other	Truck presence	
Urban intersections-signalized	Geometric design	Left-turn lane Number of lanes	Right-turn lane Lane width
Urban intersections-unsignalized	Geometric design	Left-turn lane Number of lanes Shoulder width	Right-turn lane Lane width Median presence

Note:
1 - TWLTL: two-way left-turn lane.

EMPIRICAL BAYES ADJUSTMENT

This part of the chapter describes how the expected crash frequency obtained from a safety prediction model can be modified to include information about the crash history for a project location. Inclusion of this information in the analysis increases the accuracy of the estimated expected crash frequency.

Need for Adjustment

Consider a segment of highway for which the ADT, length, geometry, and traffic control is known. Equation 1 can be used to estimate the expected crash frequency for the typical segment (with similar properties) before any changes are made to its geometry (= $E[N]$). A change in geometry is proposed and Equation 1 is used a second time to estimate the expected crash frequency for the typical segment (with similar properties) after the change (= $E[N]_{after}$). The difference between these two values is a *good* estimate of the expected change in crash frequency for the subject segment (i.e., $\Delta N = E[N]_{after} - E[N]$).

Now consider the situation where the crash history is available for the subject highway segment. This crash history indicates that X crashes were reported for the previous y years. The ratio X/y represents the average crash frequency for this segment. This average is similar to the value that was obtained from Equation 1, identified previously as $E[N]$, but they will rarely be equal because (1) the subject segment will have one or two unique properties that distinguish it from the typical segment and (2) the reported crash frequency X is a random variable and it may be randomly high (or low) during the y years. Thus, the expected crash frequency for the subject segment $E[N|X]$ is obtained by taking a weighted average of $E[N]$ and X/y. To complete the analysis, Equation 1 is used a second time to estimate $E[N]_{after}$ (reflecting conditions after the change). Then, the value $E[N|X]$ is multiplied by the ratio $E[N]_{after} / E[N]$ to obtain the best estimate of the expected crash frequency for the subject segment after the change $E[N|X]_{after}$. The product of this additional analysis effort is a *better* estimate of the expected change in crash frequency for the subject segment (i.e., $\Delta N = E[N|X]_{after} - E[N|X]$).

The empirical Bayes (EB) adjustment is applicable if the facility component (i.e., road segment, intersection, or interchange ramp) being evaluated is not undergoing major physical changes (e.g., changes to its basic number of through lanes, being relocated as part of a major highway realignment project, changes in the number of intersection legs, etc.). The reason for this restriction is that the available crash history is not likely to be representative of a newly constructed highway facility that undergoes major changes.

In summary, if a facility component is not undergoing major physical changes and if its crash history is available for a recent two to three year period, then the aforementioned adjustments will increase the accuracy of the estimated expected crash frequency. The nature of the adjustment procedure is described in the next section.

Adjustment Procedure

Methods developed by Hauer (*4*) form the basis for the adjustment procedure. The procedure consists of a series of calculations that are used to establish the values of the variables on the right-hand side of Equation 8. Once established, Equation 8 can be used to obtain the EB-adjusted estimate of a facility component's expected crash frequency.

$$E[N|X] = \frac{E[N]}{E[N]_t} E[N|X]_t \tag{8}$$

where,

$E[N|X]$ = expected crash frequency for the analysis period given that X crashes were reported, crashes/yr;

$E[N]$ = expected crash frequency for the analysis period based on ADT and geometry present during this period, crashes/yr;

$E[N]_t$ = expected crash frequency for the previous time period t based on ADT and geometry present during this period, crashes/yr; and

$E[N|X]_t$ = expected crash frequency for the previous time period t given that X crashes were reported in this period, crashes/yr.

To apply Equation 8, the safety prediction model (i.e., Equation 1) is applied twice; once to estimate $E[N]$ and a second time to estimate $E[N]_t$. The first estimate is based on the ADT and geometric conditions present during the analysis period (typically the current year). The second estimate is based on the ADT and geometric conditions present during the time period t corresponding to the crash data.

Equation 8 is used to account for any time lag that may exist between the analysis period and that corresponding to the crash history. Crash data obtained from agency databases often correspond to a time period that occurs several years prior to the analysis period. As a minimum, traffic volumes are likely to have changed since this earlier time period. Other changes in geometry may also have occurred. The effects of these changes on the expected crash frequency estimate are incorporated into Equation 8 using the ratio $E[N] / E[N]_t$.

Equation 8 is also used to estimate the expected crash frequency for the subject facility component after any proposed changes to the component's geometry are implemented. In this application, Equation 1 is used to estimate the expected crash frequency after the change $E[N]_{after}$. This value is then substituted for $E[N]$ in the numerator of Equation 8, and this equation is used to estimate the expected crash frequency for the subject component after the change (= $E[N|X]_{after}$).

The variable $E[N|X]_t$ is estimated using the following equations:

$$E[N|X]_t = E[N]_t \times weight + \frac{X}{y} \times (1 - weight) \quad \textbf{(9)}$$

with,

$$weight = \left(1 + \frac{E[N]_t\, y}{k\, L}\right)^{-1} \quad \textbf{(10)}$$

where,

X = reported crash count for y years, crashes;

y = time interval during which X crashes were reported, yr;

k = over-dispersion parameter, mi^{-1};

L = highway segment length, mi; and

$weight$ = relative weight given to the prediction of expected crash frequency.

When Equation 10 is applied to an intersection or interchange ramp, the variable for segment length L is equal to 1.0.

The EB adjustment procedure cannot be used with the base models described in the *Workbook* because the corresponding over-dispersion parameter is not known for these models. However, this adjustment can be used with the calibrated models described in Chapter 4 of this report because the over-dispersion parameter was derived as part of the re-calibration process.

CHAPTER 3. ANALYSIS PROCEDURES

OVERVIEW

This chapter describes a procedure for estimating the safety of a roadway facility component (i.e., roadway segment, intersection, or interchange ramp). The procedure consists of a series of steps that, when completed, yield an estimate of the expected crash frequency for the component. The procedure includes additional steps that facilitate the analysis of a roadway section that is comprised of one or more segments or intersections. The procedure is derived largely from a similar procedure developed by Harwood et al. (*1*).

Also described in this chapter is a procedure for defining a roadway segment. The segments obtained from this procedure are intended to be homogeneous in the sense that their traffic, geometric, and traffic control device characteristics are consistent for the length of the segment.

SAFETY PREDICTION PROCEDURE

This part of the chapter describes the six steps that comprise the safety prediction procedure. The steps are outlined in Table 2 and summarized thereafter. At the completion of the procedure, the analyst will have an estimate of the expected crash frequency for a facility component and, if desired, for all components that comprise a roadway section. This procedure would be repeated for each design alternative being considered and the results used to determine the corresponding safety benefit of each alternative, relative to the existing condition.

Table 2. Overview of the Safety Prediction Procedure.

Step	Title	Description
1	Identify roadway section	Define the limits of the roadway section of interest.
2	Divide section into separate facility components	Divide the project into homogenous roadway segments, intersections, or ramps. Select one component for analysis.
3	Gather data for subject component	Collect data describing the traffic, geometry, and traffic control devices on the subject component. Acquire crash history if available and relevant.
4	Compute expected crash frequency	Use the safety prediction model to estimate the expected crash frequency for the subject facility component.
5	Repeat Steps 3 & 4 for another facility component	Repeat the analysis for each facility component in the roadway section.
6	Aggregate results for roadway section	Sum the expected crash frequency for each facility component in the section.

The procedure is sufficiently general that it can be applied to any facility component. For those steps where the analysis varies by facility component, the procedure provides specific guidance for each component.

Step 1 - Identify Roadway Section

During this initial step, the analyst should identify the limits of the roadway section of interest. The section would consist of one or more facility components. The section may be represented by the limits of a design project, or it could be a portion of the highway with a possible safety issue or concern.

Step 2 - Divide Section into Facility Components

If needed, the section identified in Step 1 is divided into facility components during this step. Each intersection and interchange ramp is defined to be a component and should be separately analyzed. If the component of interest is a length of roadway, then the roadway must be divided into homogeneous segments. In this situation, each segment is individually analyzed in Steps 3 and 4. A procedure for dividing the roadway into homogeneous segments is described in the next part of this chapter.

Step 3 - Gather Data for Subject Component

During this step, data that describe the traffic, geometry, and traffic control devices on the subject component for the analysis year are collected. The type of data needed will vary, depending on the component type and corresponding base model and associated AMFs. The analyst should consult the relevant chapter of the *Workbook* (*2*) to identify the appropriate models and AMFs as well as the associated input variables.

If the subject facility component exists and the empirical Bayes adjustment is desired, then the analyst will need to acquire the crash history for the component. Also, the analyst will need to identify the time period corresponding to the crash data and then collect the traffic, geometry, and traffic control device data for this time period as well.

If the subject component is an intersection and ADT is available for both opposing legs of the intersection, then the two ADTs can be averaged and this average used in the base model.

Step 4 - Compute Expected Crash Frequency

During this step, the analyst uses the safety prediction model (as provided in the *Workbook*) to estimate the expected crash frequency for the subject component. If the empirical Bayes-adjusted estimate is desired, then Equations 8, 9, and 10 should also be used.

Step 5 - Repeat Steps 3 and 4 for Another Component

If additional facility components on the same roadway section are also being considered during this evaluation, then Steps 3 and 4 should be repeated for each component. One estimate of expected crash frequency should be obtained for each component.

Step 6 - Aggregate Results for Roadway Section

If all of the components in a roadway section have been analyzed, then the expected crash frequency for each component should be added together, as shown previously in Equation 2. The estimate obtained in this manner represents an estimate of the expected crash frequency for the roadway section.

SEGMENTATION PROCEDURE

The analysis of a length of roadway requires that the roadway be divided into homogenous segments. In this regard, segments are homogeneous when their traffic, geometry, and traffic control device characteristics are effectively the same for the length of the segment. The following steps outline the procedure for segmenting a roadway section.

The segmentation procedure described in this part of the chapter does not apply to frontage-road segments. A frontage-road segment is defined as the length of frontage-road between two crossroads. Each segment begins with the frontage-road/crossroad terminal. The crashes at these terminals, and the ramp/frontage-road terminals, are not estimated by the frontage-road segment safety prediction model.

Step 1 - Define Initial Segments

For the roadway section of interest, as identified in Step 1 of the Safety Prediction Procedure, obtain the necessary traffic and geometry information to apply the Segmentation Rules identified in Table 3. These rules represent conditions where a new segment *must* begin. Segment boundaries are defined by these rules because the associated design element is known to have a significant effect on safety. In contrast, the Subdivision Criteria listed in Table 3 (and discussed in Step 3) represent conditions where a new segment *should* begin. These criteria are used to identify situations where the change in design element is sufficiently significant that it may have an effect on safety.

To define the initial segments, the analyst should proceed from the start of the roadway section and work to the end of the section. The first segment would start with the beginning of the section and end when one or more of the rules in Table 3 indicates the need to start a new segment.

It should be noted that an intersection does not necessarily define the boundary of a segment. One or more intersections may be located on a segment. The segment safety prediction model estimates only those crashes that are "segment-related." These crashes do not include crashes that were identified on the crash report as being "at intersection" or "intersection-related." At-intersection and intersection-related crashes are always separately estimated using the intersection safety prediction model. Although it is not necessary to define the beginning of a new segment at an intersection, it may be necessary to do so if the ADT changes at the intersection by an amount that exceeds the Rule specified for ADT in Table 3.

Table 3. Segmentation Rules and Subdivision Criteria.

Category	Design Element	Rule or Criterion
Segmentation Rules	ADT	Segment if change exceeds ± 5 percent.
	Horizontal Curvature	Segment at beginning and end of curve (or spiral, if present). Include spiral transitions as part of the length of the curve.
	Centerline Rumble Strips	Segment at beginning and end.
	Two-way left-turn lane	Segment at beginning and end.
	Passing lane	Segment at beginning and end.
Segment Subdivision Criteria	Roadway grade [1]	Subdivide at point of change if change ≥3 percent.
	Lane width	Subdivide at point of change if change ≥ ± 1 ft.
	Outside shoulder width	Subdivide at point of change if change ≥ ± 1 ft.
	Horizontal clearance	Subdivide at point of change if change ≥ ± 10 ft.
	Side slope	Subdivide at point of change if slope changes from one of the following categories to another category: ● 1:3 or flatter ● steeper than 1:3
	Utility pole offset	Subdivide at point of change if offset changes from one of the following categories to another category: ● 10 ft or greater ● 5 ft to 10 ft ● 2 ft to 5 ft ● 0 ft to 2 ft
	Relative bridge width	Subdivide at the beginning and end of a narrow bridge if the bridge width is less than the traveled-way width plus 2 ft.

Note:
1 - If a segment is subdivided because of grade change, the new segment should begin at the point of inflection (PI).

Step 2 - Adjust Length of Short Segments

All segments defined in Step 1 should have a length of 0.1 mi or more due to the precision of crash location reporting in Texas. If a design element listed in Table 1 is used to define a segment but the length of the element is less than 0.1 mi (e.g., a horizontal curve of 0.07 mi), then the segment length should be increased to 0.1 mi. Any segment that has its length increased in this manner should be centered on the short design element. The extra length for this segment would come from the adjacent roadway segments. The effective length of the design element for AMF calculation would be 0.1 mi (not the actual length). This concept is illustrated in Figure 1 where the segment containing a 0.07 mi curve has its length increased to 0.1 mi.

Step 3 - Define Additional Segments

For all non-curved segments, apply the Subdivision Criteria identified in Table 3 to determine if further subdivision of the initial segments is needed. As noted in Step 2, all segments should have a length of 0.1 mi or more due to the precision of crash location reporting in Texas. If, as a result of applying the subdivision criteria, a segment is less than 0.1 mi long, then it should be combined with an adjacent non-curved segment, if possible, to create a new segment that is at least 0.1 mi long.

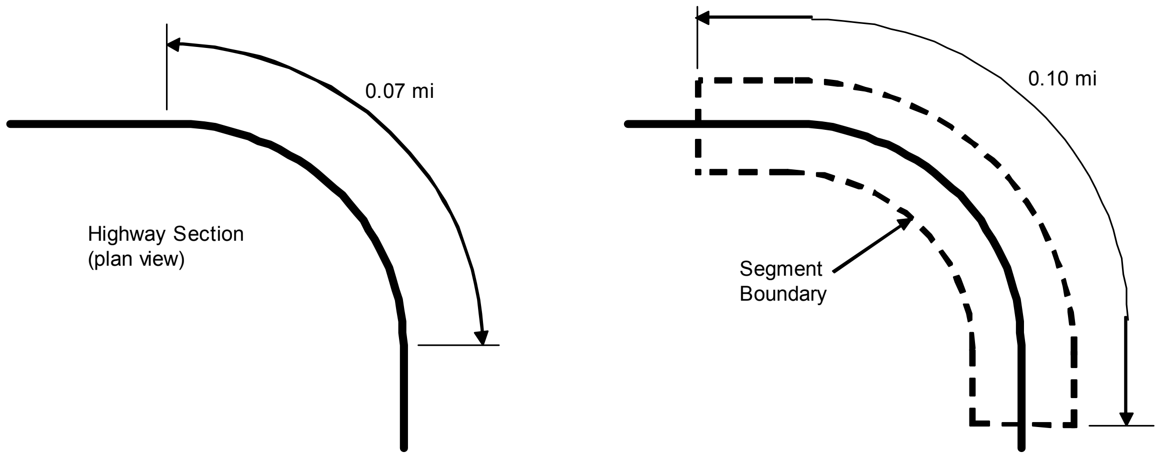

Figure 1. Illustration of Segment Definition for Short Design Elements.

Step 4 - Define Segments Based on Judgment

If, after completing Steps 1, 2, and 3, the combination of changes in one or more design elements in any one segment is believed likely to result in a net change in the roadway character, then engineering judgement should be used to decide whether or not to further subdivide the segment to accommodate these changes. The changes considered can be in the design elements listed in Table 3 as well as any other elements of the roadway or roadside environment believed to have an influence on safety (e.g., driveway density, presence of shoulder rumble strips, etc.).

Supplemental Guidelines

Any design element dimension (e.g., lane width) that varies within a segment should be averaged, and the average used as input to the appropriate AMF or base model. A variation of this guidance applies when grade changes within a section. Specifically, if grade varies along a segment and the change in grade at the point of inflection (PI) does not exceed 3.0 percent, then the value of grade used in the grade AMF should equal the weighted average of the absolute value of the vertical alignment tangent grades along the segment, where the weight used is the length of the tangent. This variation is illustrated by example in Figure 2.

The vertical alignment of a roadway is shown in Figure 2. The grade change of 4.5 percent at the crest curve exceeds 3.0 percent. According to the segmentation subdivision criteria in Table 3, segment A should end and Segment B should begin at the PI of this curve. The grade to use in the grade AMF for segment A should equal that of the vertical alignment tangent, which is 3.0 percent. The grade change at the sag curve does not exceed 3.0 percent, so a new segment does not need to begin at this PI. However, at a point 300 ft beyond this PI, a change in cross section requires segment B to end. According to the aforementioned guidance, the grade to use in the grade AMF for this segment is 1.3 percent. This value is obtained as the length-weighted average of the two grades (as shown in the figure). It should be noted that the absolute value of each grade is used in the calculation.

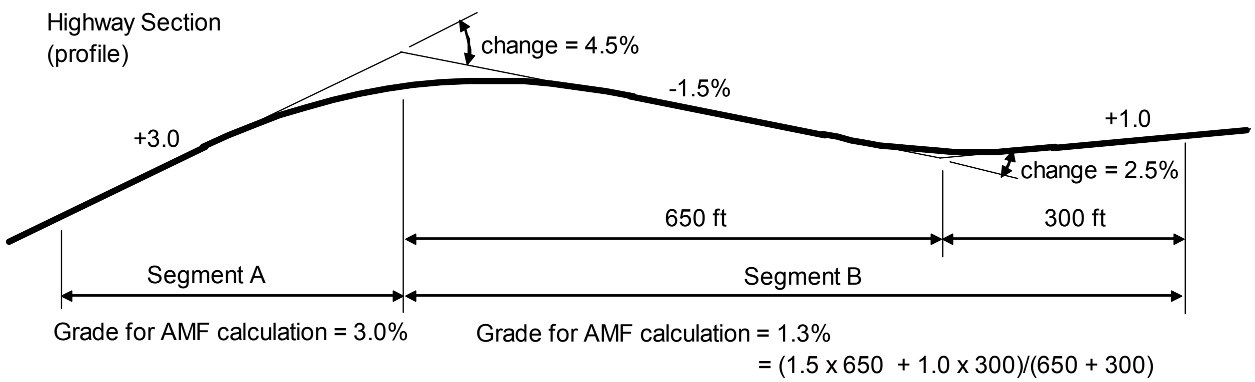

Figure 2. Example Grade Calculation.

CHAPTER 4. SUPPLEMENTAL MODELS

OVERVIEW

This chapter summarizes safety evaluation tools developed subsequent to the publication of the *Workbook* (*2*). The summary is provided in the two main parts of the chapter. The first part summarizes the tools developed for rural frontage-road segments. The second part summarizes the tools developed for rural two-lane highways. The tools developed include base models and AMFs. The report by Bonneson et al. (*5*) documents the methods used to develop these tools. It also provides the findings from a sensitivity analysis as well as their comparison with similar models and AMFs reported in the literature.

RURAL FRONTAGE ROADS

This part of the chapter describes the development of quantitative tools for evaluating the safety of rural frontage-road segments in Texas. These tools include a base model for estimating the crash frequency of typical frontage-road segments and two AMFs. These tools do not address the safety of ramp/frontage-road terminals or the safety of frontage-road/crossroad intersections. Moreover, they do not directly address the safety of one-way frontage-road operation versus two-way frontage-road operation.

Base Model

This section describes the base model for rural frontage-road segments. A frontage-road segment is defined as the length of frontage-road between two crossroads. The model is applicable to segments with either one-way or two-way operation. Crashes that occur at the frontage-road/crossroad terminals and at the ramp/frontage-road terminals are not included in the estimate obtained from the base model. The base model for frontage roads is:

$$E[N]_b = 0.00134\ ADT^{0.641}\ L \quad \textbf{(11)}$$

where,

$E[N]_b$ = expected base crash frequency, crashes/yr;
ADT = average daily traffic volume, veh/d; and
L = segment length, mi.

Equation 11 predicts the crash frequency that would be estimated for a frontage-road segment with 12 ft lanes and an average paved shoulder width of 1.5 ft. In application, the crash frequency predicted by Equation 11 would be multiplied by the AMFs for lane width and shoulder width described in the next section to estimate the crash frequency for a given segment with a specified lane and shoulder width.

If this base model is used to obtain an EB-adjusted estimate of expected crash frequency, the over-dispersion factor k to use for this purpose has a value of 1.37 mi^{-1}.

Equation 11 is compared in Figure 3 with the rural two-lane highway base model included in the *Workbook*. The *Workbook* model is based on an injury (plus fatal) crash rate of 0.20 cr/mvm. The trend lines in the figure indicate that a frontage road experiences slightly more injury (or fatal) crashes than a rural two-lane highway for ADTs less than 3500 veh/d. The reverse trend applies for ADTs greater than 3500 veh/d. It is possible that the increased turning and weaving activity associated with the frontage road (relative to the two-lane highway) may explain the slightly higher crash frequency on frontage roads for ADTs less than 3500 veh/d. As ADT exceeds 3500 veh/d, there may be less opportunity for turning (i.e., fewer gaps) and the weaving activity may be more constrained (i.e., lower speed) on the frontage road, such that frontage-road crash frequency is lower than that found on two-lane highways.

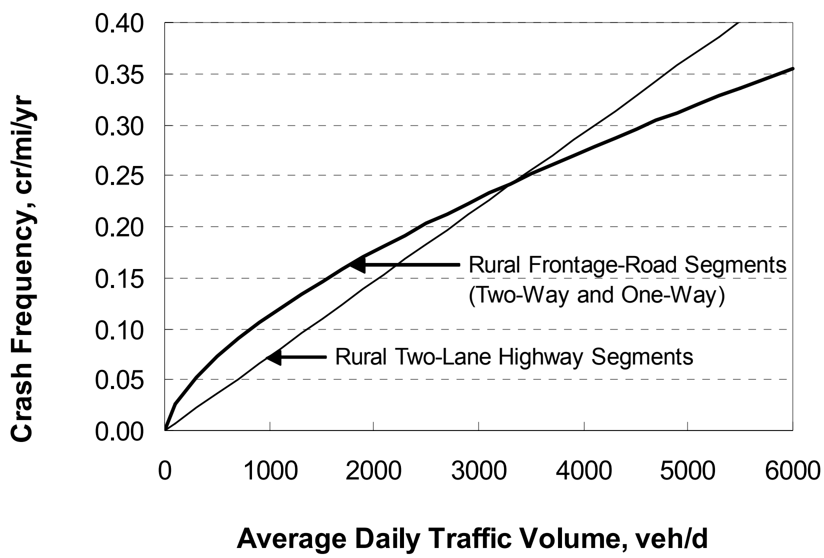

Figure 3. Comparison between Frontage-Road Segment Model and Rural Two-Lane Highway Segment Model.

Accident Modification Factors

This section describes two AMFs that were derived from the frontage-road segment crash data. One AMF describes the relationship between lane width and crash frequency. The second AMF describes the relationship between shoulder width and crash frequency.

Lane Width AMF

The recommended AMF for frontage-road lane width is:

$$AMF_{LW} = e^{-0.188\,(W_l - 12.0)} \qquad \textbf{(12)}$$

where,

AMF_{LW} = lane width accident modification factor; and

W_l = average lane width, ft.

The average lane width used in Equation 12 represents the total width of all through traffic lanes on the frontage road divided by the number of through lanes. The value of 12.0 in Equation 12 reflects the base, or typical, lane width condition. By definition, it is associated with an AMF value of 1.0.

The graphical representation of the lane width AMF is shown in Figure 4. The relationship between lane width and AMF value shown in this figure suggests that crash frequency is reduced about 17 percent for a 1 ft increase in lane width. Based on the range of lane widths in the database, the lane width AMF is applicable to lane widths ranging from 9 to 12 ft.

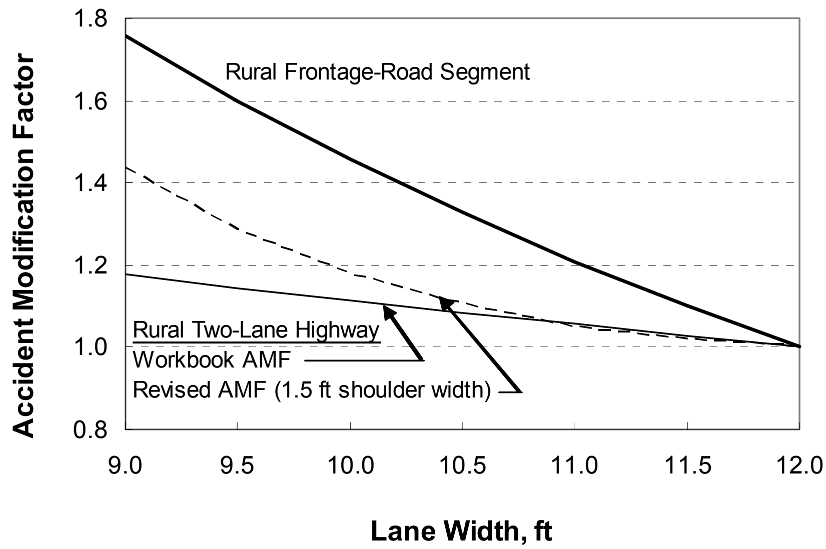

Figure 4. AMF for Frontage-Road Lane Width.

Also shown in Figure 4 is the lane width AMF for rural two-lane highways from the *Workbook* as well as the revised lane width AMF described in the next part of this chapter. A comparison of these AMFs with the lane width AMF for frontage roads suggests that lane width on a frontage road has a greater impact on crash frequency than it does on a two-lane highway. It is possible that this trend stems from the relatively high percentage of turning traffic and the considerable weaving activity that occurs on frontage roads (between the ramp terminals and the crossroad intersection), relative to a two-lane highway. Wider lanes on frontage-road segments may provide some additional room for recovery when these turning and weaving-related conflicts occur.

Shoulder Width AMF

The recommended AMF for frontage-road shoulder width is:

$$AMF_{SW} = e^{-0.070\,(W_s\,-\,1.5)} \qquad \textbf{(13)}$$

where,
AMF_{SW} = shoulder width accident modification factor;

W_s = average paved shoulder width (= $[W_{s,r} + W_{s,l}]/2$), ft;
$W_{s,r}$ = paved shoulder width on the right side of the frontage road, ft; and
$W_{s,l}$ = paved shoulder width on the left side of the frontage road, ft.

The average paved shoulder width used in Equation 13 represents the average of the left- and right-side shoulder widths. The value of 1.5 in Equation 13 reflects the base, or typical, average shoulder width condition for frontage roads. By definition, it is associated with an AMF value of 1.0.

The graphical representation of the shoulder width AMF is shown in Figure 5. The relationship between frontage-road shoulder width and the AMF value suggests that crash frequency is reduced about 7 percent for a 1 ft increase in average shoulder width. Based on the range of shoulder widths in the database, the AMF is applicable to shoulder widths ranging from 0 to 5 ft.

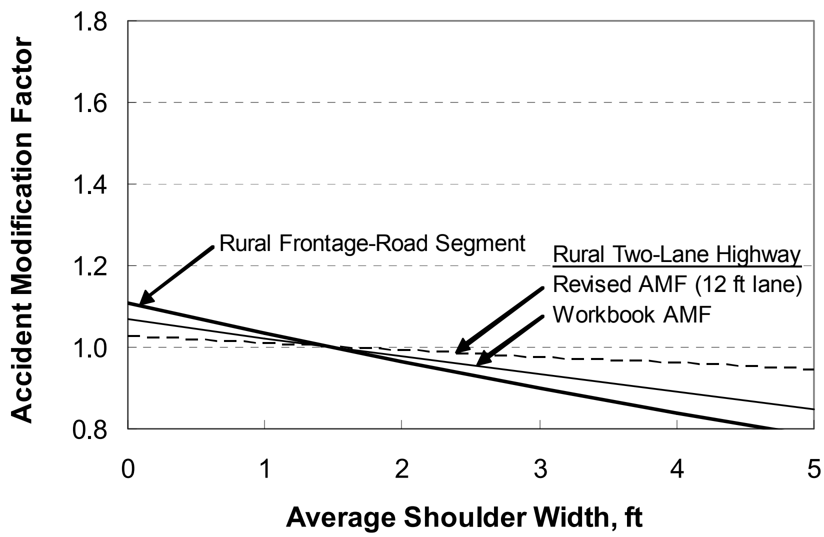

Figure 5. AMF for Frontage-Road Shoulder Width.

Also shown in Figure 5 is the shoulder width AMF for rural two-lane highways from the *Workbook* as well as the revised shoulder width AMF described in the next part of this chapter. The base condition for the *Workbook* AMF has been changed to a shoulder width of 1.5 ft to facilitate its comparison with Equation 13. As suggested by the trend lines in this figure, shoulder width has a slightly larger impact on frontage-road safety than on rural two-lane highways. This trend is consistent with that found for the lane width AMF.

RURAL TWO-LANE HIGHWAYS

This part of the chapter summarizes the findings from research undertaken to re-calibrate the existing safety prediction models for rural two-lane highways and intersections in Texas. The existing models considered for re-calibration are described in the *Workbook*. The models that are summarized in this part include:

- highway segments (excluding intersections);
- three-leg intersections with two-way stop control;
- four-leg intersections with two-way stop control; and
- four-leg intersections with signal control.

Each model provides an estimate of the expected injury (plus fatal) crash frequency for the associated component, given specified traffic volume and geometric design conditions. However, each model was initially calibrated using data from locations outside of Texas. To ensure that the estimate is not biased, the model was re-calibrated using data specific to Texas.

In the first section to follow, re-calibrated base models are described for each of the aforementioned facility components. The second section describes AMFs for curve radius, lane width, and shoulder width.

Base Models

Two-Lane Highway Segment

The re-calibrated base model for rural two-lane highway segments is:

$$E[N]_b = 0.0537 \left(\frac{ADT}{1000}\right)^{1.30} L \tag{14}$$

where,

$E[N]_b$ = expected base crash frequency, crashes/yr;
ADT = average daily traffic volume, veh/d; and
L = highway segment length, mi.

Equation 14 predicts the crash frequency that would be estimated for a highway segment with 12 ft lanes, an 8 ft paved shoulder width, and other geometric conditions considered typical in Texas. In application, the crash frequency predicted by Equation 14 would be multiplied by the AMFs in Chapter 3 of the *Workbook* (except those replaced by the AMFs described in the next section) to estimate the crash frequency for a given segment with specified geometric and traffic characteristics.

If this base model is used to obtain an EB-adjusted estimate of expected crash frequency, the over-dispersion factor k to use for this purpose has a value of 15.3 mi^{-1}.

The relationship between segment crash frequency and daily traffic volume is shown in Figure 6. The thick trend line corresponds to the re-calibrated model (i.e., Equation 14). The thin dashed trend line corresponds to the existing base model in the *Workbook*.

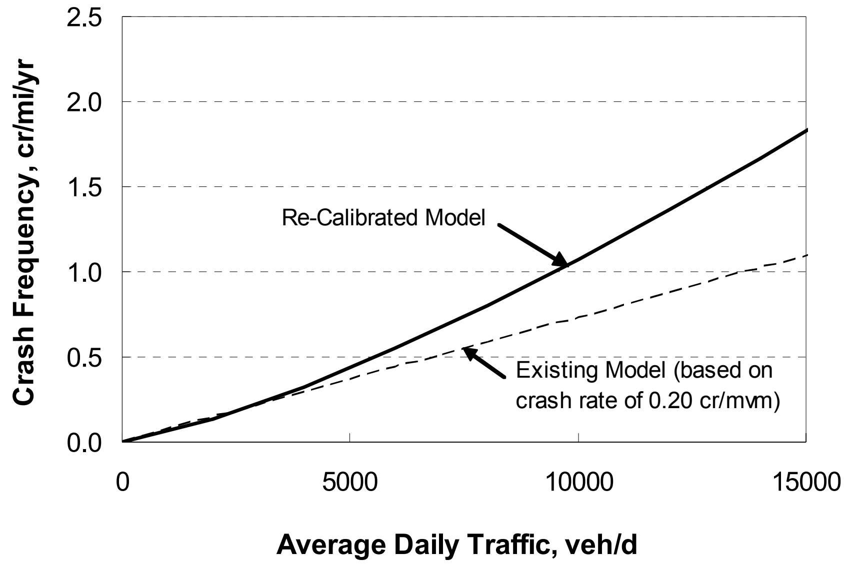

Figure 6. Relationship between Volume and Segment Crash Frequency.

The two trend lines in Figure 6 indicate that the re-calibrated model and the existing model yield similar estimates of crash frequency for ADTs of 5000 veh/d or less. For ADTs above 5000 veh/d, the estimated crash frequency from the re-calibrated model exceeds that obtained from the existing model.

Three-Leg Unsignalized Intersection

The re-calibrated base model for three-leg unsignalized intersections is applicable to those intersections on rural two-lane highways that have stop-control on the minor roadway. The form of this model is:

$$E[N]_{3LST} = 0.0973 \left(\frac{ADT_{major}}{1000} \right)^{0.863} \left(\frac{ADT_{minor}}{1000} \right)^{0.497} \qquad \textbf{(15)}$$

where,

$E[N]_{3LST}$ = expected crash frequency for a three-leg unsignalized intersection, crashes/yr;
ADT_{major} = average daily traffic volume on the major road, veh/d; and
ADT_{minor} = average daily traffic volume on the minor road, veh/d.

Equation 15 predicts the crash frequency that would be estimated for a three-leg intersection having geometric conditions considered typical in Texas. In application, the crash frequency predicted by Equation 15 would be multiplied by the AMFs in Chapter 6 of the *Workbook* to estimate the crash frequency for a given segment with specified geometric and traffic characteristics.

If this base model is used to obtain an EB-adjusted estimate of expected crash frequency, the over-dispersion factor k to use for this purpose has a value of 2.59.

The re-calibration process for this model indicated that the re-calibrated model coefficients were not significantly different from those in the existing model in the *Workbook*. Thus, the existing model was retained. Equation 15 yields the same expected crash frequency as the existing model.

Four-Leg Signalized Intersection

The re-calibrated base model for four-leg signalized intersections is applicable to intersections on rural two-lane highways. The form of this model is:

$$E[N]_{4LSG} = 0.221\left(\frac{ADT_{major}}{1000}\right)^{0.611}\left(\frac{ADT_{minor}}{1000}\right)^{0.595} \qquad \textbf{(16)}$$

where,

$E[N]_{4LSG}$ = expected crash frequency for a four-leg signalized intersection, crashes/yr.

Equation 16 predicts the crash frequency that would be estimated for a four-leg intersection having geometric conditions considered typical in Texas. In application, the crash frequency predicted by Equation 16 would be multiplied by the AMFs in Chapter 6 of the *Workbook* to estimate the crash frequency for a given segment with specified geometric and traffic characteristics.

If this base model is used to obtain an EB-adjusted estimate of expected crash frequency, the over-dispersion factor k to use for this purpose has a value of 3.15.

The relationship between intersection crash frequency and daily traffic volume is shown in Figure 7. The thick trend line corresponds to the re-calibrated model (i.e., Equation 16). The thin dashed trend line corresponds to the existing base model in the *Workbook*.

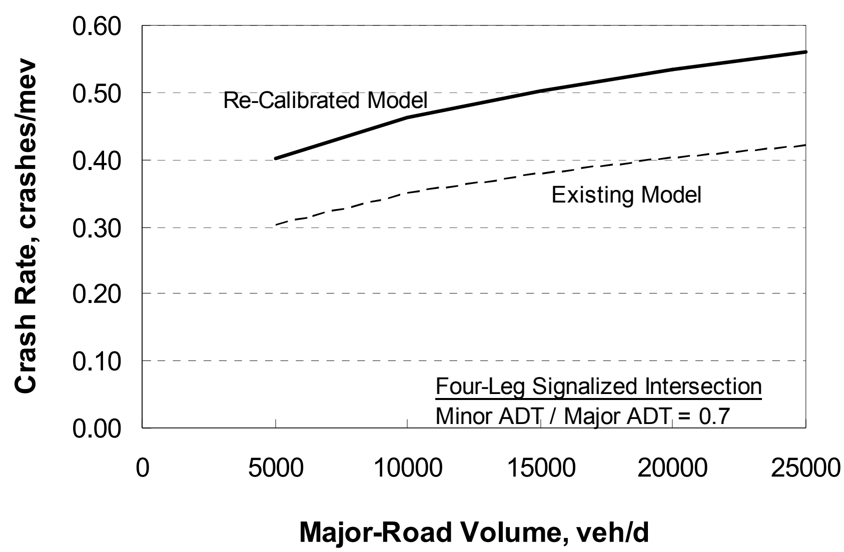

Figure 7. Relationship between Volume and Signalized Intersection Crash Frequency.

The two trend lines in Figure 7 indicate that the re-calibrated model estimates larger crash rates than the existing model yield for the full range of major-road volume. This difference is significant and illustrates the benefit of calibrating models developed using data from other states to local conditions.

Four-Leg Unsignalized Intersection

The re-calibrated base model for four-leg unsignalized intersections is applicable to those intersections on rural two-lane highways that have two-way stop control. The form of this model is:

$$E[N]_{4LST} = 0.235\left(\frac{ADT_{major}}{1000}\right)^{0.692}\left(\frac{ADT_{minor}}{1000}\right)^{0.514} \quad \textbf{(17)}$$

where,

$E[N]_{4LST}$ = expected crash frequency for a four-leg unsignalized intersection, crashes/yr.

Equation 17 predicts the crash frequency that would be estimated for a four-leg intersection having geometric conditions considered typical in Texas. In application, the crash frequency predicted by Equation 17 would be multiplied by the AMFs in Chapter 6 of the *Workbook* to estimate the crash frequency for a given segment with specified geometric and traffic characteristics.

If this base model is used to obtain an EB-adjusted estimate of expected crash frequency, the over-dispersion factor k to use for this purpose has a value of 1.61.

The relationship between intersection crash frequency and daily traffic volume is shown in Figure 8. The thick trend line corresponds to the re-calibrated model (i.e., Equation 17). The thin dashed trend line corresponds to the existing base model in the *Workbook*.

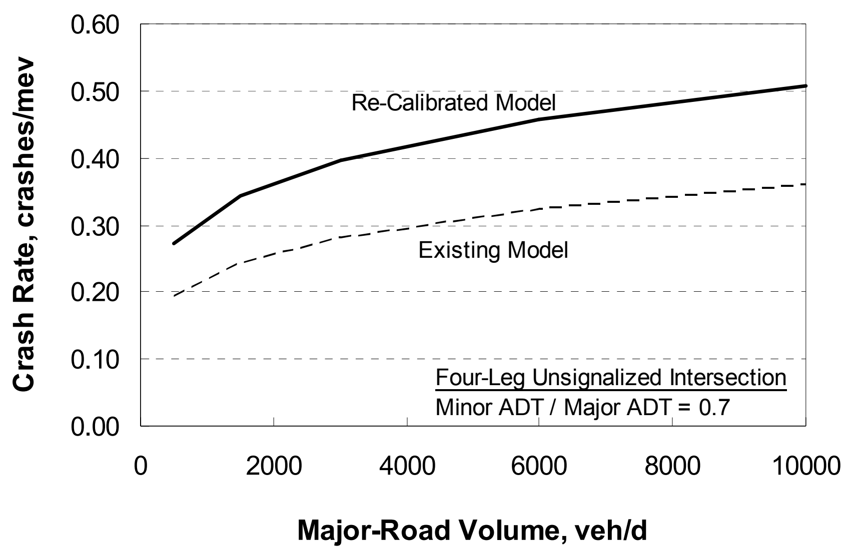

Figure 8. Relationship between Volume and Unsignalized Intersection Crash Frequency.

The two trend lines in Figure 8 indicate that the re-calibrated model estimates larger crash rates than the existing model yield for the full range of major-road volume. This difference is significant and illustrates the benefit of calibrating models developed using data from other states to local conditions.

Accident Modification Factors

This section describes two AMFs that were derived from using crash data for rural two-lane highway segments in Texas. One AMF describes the relationship between horizontal curve radius and crash frequency. A second AMF describes a relationship between lane width, shoulder width, and crash frequency.

Horizontal Curve Radius AMF

The recommended AMF for horizontal curve radius is:

$$AMF_{cr} = 1 + 0.106 \left(\frac{L_c}{L} \right) \left(\frac{5730}{R_c} \right)^2 \quad \textbf{(18)}$$

where,

AMF_{cr} = horizontal curve radius accident modification factor;
L_c = horizontal curve length, mi; and
R_c = curve radius, ft.

This AMF replaces the horizontal curve AMF in Chapter 3 of the *Workbook*. The radius used in Equation 18 corresponds to the radius of the roadway centerline. The AMF applies only to circular curves and the circular portion of curves with spiral transitions. The AMF for spiral transition presence provided in the *Workbook* should be used with this AMF if spiral transitions are present. The AMF converges to a value of 1.0 as the radius approaches infinity. Thus, the base, or typical, condition is a tangent highway section.

The ratio of curve length L_c to segment length L is included in Equation 18 to allow it to be used on segments that are longer than the length of the curve. This situation can occur when the curve length is less than 0.1 mi and the segment length is 0.1 mi (as noted in Chapter 3, the minimum segment length is 0.1 mi). If the curve is less than 0.1 mi, then the segment length L is set equal to 0.1 mi. In this situation, the length ratio in Equation 18 adjusts the AMF such that it does not overestimate the effect of the curve, relative to the full length of the segment.

The recommended revised AMF model is illustrated in Figure 9. The values obtained from this model are shown with a solid trend line. The values obtained from the horizontal curve AMF in the *Workbook* are shown using two dashed lines. The revised AMF is shown to indicate a larger AMF value for all radii, relative to the *Workbook* AMF.

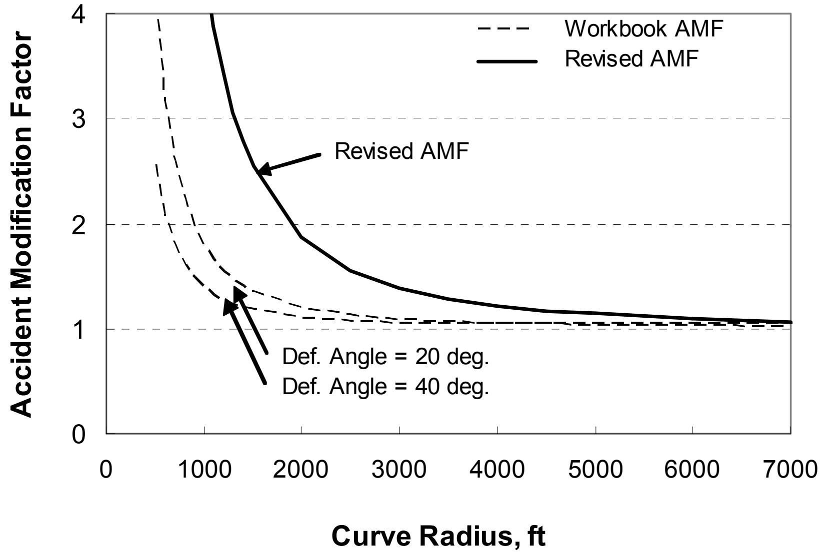

Figure 9. AMF for Two-Lane Highway Curvature.

Combined Lane Width and Shoulder Width AMF

The recommended AMF for combined lane width and shoulder width is:

$$AMF_{lw,osw} = \left(e^{0.235\ +\ 0.0533\ [W_l - 12.5]^2\ -\ 0.163\ W_s\ +\ 0.011\ W_s W_l}\ -\ 1\right) 0.54 + 1.0 \qquad \textbf{(19)}$$

where,

$AMF_{lw,\ osw}$ = lane width and shoulder width accident modification factor;
W_l = lane width, ft; and
W_s = shoulder width.

This AMF replaces the lane width and outside shoulder width AMFs in Chapter 3 of the *Workbook*, as applied to two-lane highways. The AMF in Equation 19 replaces both the lane width and the shoulder width AMFs in the *Workbook*. It includes both variables in a single AMF because of an interaction that was found between lane width and shoulder width. Specifically, it suggests that the effect of lane width on safety is lessened on roadways with wider shoulder widths, which is logical. The AMF reflects a base condition of 12 ft lane width and 8 ft shoulder width. By definition, a segment with this lane and shoulder width will have an AMF value of 1.0.

The recommended revised AMF model is illustrated in Figure 10 for a range of lane widths. The values obtained from Equation 19 are shown with a thick trend line. The values obtained from the lane width AMF in the *Workbook* are shown with a thin line. The AMF values from the revised model with an 8ft shoulder width are comparable to those in the *Workbook*. The two thick trend lines indicate that the relationship between AMF value and lane width varies, depending on the shoulder width. Lane width is indicated to have a larger effect on safety when the shoulder is narrow, which is logical.

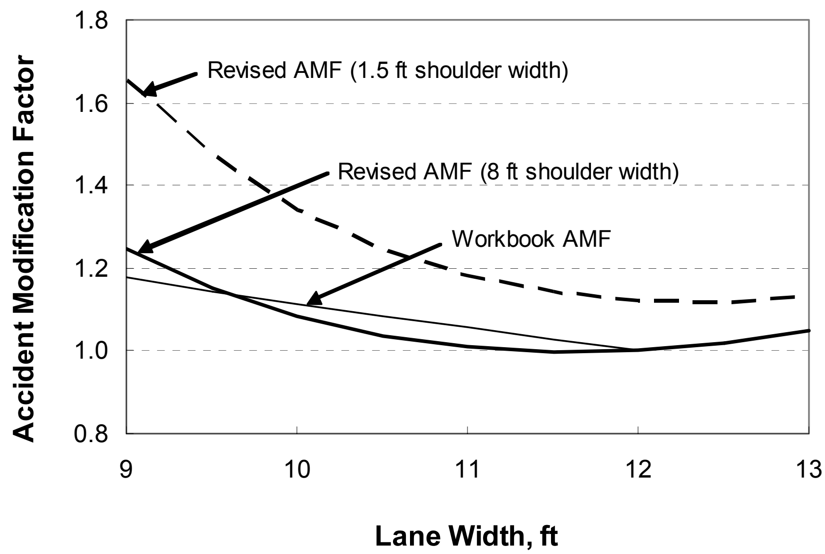

Figure 10. AMF for Two-Lane Highway Lane Width.

The revised AMF model is shown in Figure 11 for a range of shoulder widths. The values obtained from Equation 19 are shown with the two thick trend lines. The thick dashed trend line corresponds to shoulders adjacent to 10 ft lanes. The thick solid trend line corresponds to shoulders adjacent to 12 ft lanes. The values obtained from the shoulder width AMF in the *Workbook* are shown with a thin solid line.

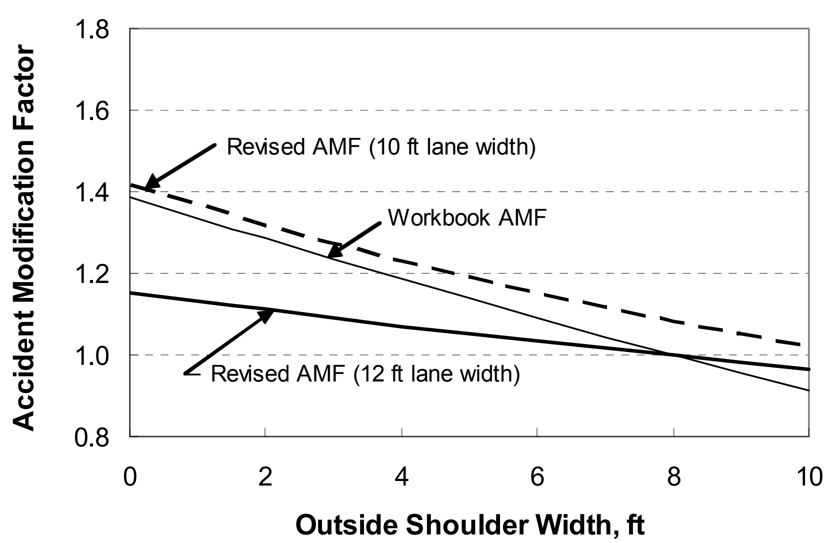

Figure 11. AMF for Two-Lane Highway Shoulder Width.

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